Report on:

SHORE PROTECTION STRUCTURES
LAKE MICHIGAN POTENTIAL DAMAGES STUDY

Prepared for:

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1.0 INTRODUCTION

This report was prepared as part of the ongoing Lake Michigan Potential Damages Study - Phase III, conducted by the NTH/WTA Joint Venture for the U.S. Army Corps of Engineers, Detroit District. This report is intended to assess the effects of changes in Lake Michigan water levels on the performance and stability of existing shore protection structures. The results of this assessment may be used by others to estimate the potential costs for the repair and maintenance of these structures for effects associated with changes in the lake water levels.

1.1 LIMITATIONS

The purpose of this evaluation is to assess potential damage to typical shore protection structures. The evaluations presented in this report should not be used for design purposes. Site-specific information should be collected and used in the design or assessment of specific structures.

1.2 BACKGROUND

The U.S. Army Corps of Engineers (USACE) Detroit District is currently conducting Phase III of Lake Michigan Potential Damages Study (LMPDS) to provide an assessment of potential shoreline damages due to changes in Lake Michigan water levels.

Changes in lake levels may affect the performance and stability of shore protection structures. High water elevations may result in overtopping and flanking while low water levels may increase scour along the toe of these structures. This report presents an assessment of the effects of changes in the lake levels on the stability of typical shoreline structures.

1.3 OBJECTIVES

The objective of this report is to develop an assessment of the potential damage to typical shoreline structures due to changing lake levels. The objectives of the assessment described in this report are as follows:

- Select typical shore protection structures
- Develop typical configurations for each selected structure
- Select potential high and low water levels for each structure
- Evaluate the effects of high and low water levels on these structures

1.4 THE STUDY AREA

The study area for this report includes structures along the shoreline of five selected counties in Michigan and Wisconsin. These counties include Ottawa, Allegan, Ozaukee, Sheboygan and Manitowoc. An inventory of shore protection types for these counties has been prepared by Orca Technologies International, Inc. (OTI) and is presented later in this report. The results of this study may ultimately be extended to estimate potential damages along the entire shore of Lake Michigan but only insofar that the structure types and shoreline conditions evaluated herein may be representative of conditions elsewhere.

2.0 SHORE PROTECTION STRUCTURES

There are many different types of structures that exist along the shoreline of Lake Michigan. For the purpose of this study, an actual detailed survey of these structures and their configurations would have been cumbersome and time consuming. As mentioned before, OTI completed a survey of the shore protection types along selected counties using aerial photographs. The shore protection types were classified in accordance with the existing Shore Protection Classification System. Based on a review of the survey information, several most common structure types were selected for evaluation in this study.

In the following sections, we present the shore protection classification system, the results of the OTI survey and the structure types selected for this evaluation.

2.1 SHORE PROTECTION CLASSIFICATION SYSTEM

The shore protection classification system, presented in a report entitled *A Revised Geomorphic, Shore Protection and Nearshore Classification of the Lake Michigan Shoreline* prepared by Vision Group International Inc. (VGI), and dated September 1998, was adapted for use in the LMPDS. This system classifies the shore protection structures into the following categories:

1. COASTAL ARMORING
   - 1a. Revetments
   - 1b. Seawalls / Bulkheads

2. BEACH EROSION CONTROL DEVICES
   - 2a. Groins
   - 2b. Jetties (littoral barriers)
   - 2c. Offshore Breakwaters
   - 2d. Perched Beaches

3. NON-STRUCTURAL
   - 3a. Beach Nourishment
   - 3b. Vegetation Planting / Bioengineering
   - 3c. Slope Grading / Bluff Stabilization

4. PROTECTED WETLANDS

5. AD-HOC
   - 5a. Concrete Rubble
   - 5b. Other Materials (tires, mats, etc.)

6. UNCLASSIFIED

7. UNPROTECTED

2.2 REVIEW OF OTI SURVEY

As part of Phase III of the LMPDS, OTI completed an inventory of the shore protection types in the 5 selected counties. The inventory was conducted using 1:6000 scale 1989 and 1999 aerial photography for Ottawa and Allegan Counties in Michigan and 1978 and 1992 aerial photography for Ozaukee, Sheboygan and Manitowoc Counties in Wisconsin. The surveyed structures were classified using the existing shoreline protection classification system. A summary of the surveyed types and their percentages is included in Appendix A.

A review of these tables indicates that there are three main types of structures along the shorelines of the selected counties: (1) revetments, (2) seawalls/bulkheads, and (3) groins. Therefore, these three structure types have been selected for our evaluation. A detailed description of the selected structures is presented in the following section.

2.3 SELECTED SHORE PROTECTION STRUCTURES

Revetments, seawalls/bulkheads and groins are the most common structures along the study area shorelines. There are several types of each of these structures, varying by shape and construction materials. Since the scope of work for this study does not allow the evaluation of all types, we selected typical types for our evaluation. A brief description of the different types of each structure, the selected type(s) and the basis for such selection are presented below.

2.3.1 Seawalls/Bulkheads

Seawalls and bulkheads are structures along the shoreline designed to resist wave forces, prevent erosion and/or retain fill. Depending on the design conditions and construction material, the face of seawall structures may have several shapes including curved, stepped and vertical. Bulkheads are usually anchored vertical pile walls or gravity walls. The design of these structures usually includes an armoring of rock/stone along the toe to minimize scour by wave action. These walls are usually constructed using concrete, steel or timber.

Based on discussions with the USACE staff in the Grand Haven Area office (Mr. Ross Kittleman, Area Engineer), the most common type of seawall/bulkhead structures in the study area is a vertical wall constructed using steel or timber sheetpiles. Therefore, this type of structure was used in our analyses.

2.3.2 Revetments

Revetments are usually designed to protect shorelines against erosion. There are two types of revetments: rigid, cast-in-place concrete and flexible or articulated armor which includes riprap or quarry-stone revetments. Discussions with the USACE's Grand Haven staff indicates that riprap revetments are the most common type of revetments found along the study area shores. Therefore, this type was used in our analyses.

2.3.3 Groins

Groins are usually thin, straight structures that are placed perpendicular to the shoreline in an attempt at reducing erosion and to trap sediments. They can be found either alone or in groups. As described in the USACE Engineering and Design Manual, entitled "Coastal Groins and Nearshore Breakwaters", the main purposes of groins are to build or widen a beach, to stabilize a beach subject to severe storms, to reduce the rate of longshore transport of sand out of an area and to reduce the accumulation of sand in a down drift area.

Although groins are usually straight and perpendicular to the shore, they are occasionally curved or T shaped. These rather different layouts result in an increase in costs and are proven not to be as effective as the more commonly used straight groin.

As indicated in the “Shore Protection Manual” published by the USACE, groins are mainly classified based on their permeability, height and length. Groins can be of different heights and lengths, and can be made of materials with different permeabilities. More specifically, they can be made of stone, concrete, timber, steel, asphalt and concrete grout-filled geotextiles. Groins can be classified as short or long. Groins that extend beyond the surf zone are considered to be long, and those that do not are considered to be short.

Based on discussions with the USACE’s staff, concrete grout-filled geotextile groins are currently the most common type of groins found in the study area. Each groin usually consists of two long tubes overlain by a third tube. These groins are usually straight and perpendicular to the shoreline. Therefore, grout-filled geotextile groins were evaluated in this study.

3.0 COASTAL DESIGN PARAMETERS

In addition to changes in lake levels, there are several coastal parameters that affect the stability of shore protection structures. These parameters, including changes in lake levels, are presented below. Other structure-specific parameters are described in Section 4.

3.1 LAKE LEVEL ELEVATION

The Lake Michigan water levels fluctuate naturally between high and low levels that have been recorded over a period of almost a century. During that period, record high and low levels of approximately 177.74 m (583 feet IGLD 1985) and 175.91m (577 feet IGLD 1985) have been recorded. The location of the lake level relative to the shore protection structure affects its stability. As described in Section 4, we selected several lake level locations, relative to the shore protection structure, to evaluate this effect.

3.2 LAKE BED SLOPE

The lake bed slope affects the potential for scour and the wave breaking height and has a minor effect on global slope stability (stability of the structure/slope usually associated with movements including rotational slope failure, sliding and overturning). Based on discussions with W. F. Baird and Associates, typical bed slopes in the study area range from 1 vertical to 10 horizontal and 1 vertical to 20 horizontal. These slopes were used in our analyses.

3.3 WAVE CHARACTERISTICS

Waves are characterized by their height and period. Higher waves and longer periods are associated with stronger forces and a higher potential for runup and overtopping. For this evaluation, for each structure, we evaluated the effect of a critical wave on the stability of the structures. The wave characteristics were provided by W.F. Baird and Associates. The height and period of the selected wave are 5.0 m and 9 seconds, respectively.

There are three basic types of waves: non-breaking, breaking and broken waves. While lake level changes may not affect the non-breaking wave characteristics, the breaking wave characteristics are dependent on the depth of water and lake bed slope. The breaking waves most often exert higher forces than non-breaking waves. Broken waves are usually associated with the smallest forces.

3.4 SOIL STRATIGRAPHY & SHEAR STRENGTH

Soil stratigraphy has a significant effect on the stability of shore protection structures. The stresses sustained by these structures and their global stability are dependent on the type of foundation and retained soils and their shear strengths. In general, cohesive soils usually apply higher loads than cohesionless soils and require special drainage design features. To represent a range of potential soil stratigraphy, we selected two typical soil profiles; a cohesive and a cohesionless soil profiles.

For the cohesive soils, short-term and long-term stability evaluations are usually performed. Since we were evaluating the stability of existing structures, we assumed that the long-term stability of these structures is more critical than the short-term stability. Therefore, only effective stress shear strength parameters were selected for analysis. We selected the following typical effective shear strength parameters: a total density of 19.2 kN/m^3 (122 pcf), a cohesion and internal friction angle of 0.0 Pa and 28°, respectively, and a friction angle between the soil and seawall/bulkheads of 10°.

For the cohesionless soils, we selected the following typical shear strength parameters: a total density of 19.2 kN/m^3 (122 pcf), an internal friction angle of 30° and a friction angle between the soil and seawalls/bulkheads of 14°.

3.5 ICE FORCES

Ice can affect marine structures in a number of ways. Moving surface ice can cause significant crushing and bending forces as well as large impact loadings. Vertical forces can be caused by the weight of ice on structures during falling water levels and by buoyant uplift caused by ice masses frozen to structural elements during rising water levels. The effects of ice loads are discussed in the revetment analysis in Section 4.3.2.2.

3.6 SAND SUPPLY

Sand was once available to the shore in adequate supply from streams and rivers and by natural erosion of coastal formations. Now development in the watershed areas and along previously eroding shores has progressed to a stage where large areas of the coast now receive little or no sand through natural geologic processes. Continued land development along both inland rivers and coastal areas has been accompanied by erosion control methods which have deprived the coastal areas of sediment formerly available through the natural erosion process. These methods reduce the amount of sand transported along the coast. Because the natural sand supply has been reduced, the erosion of the shore may become gradually more severe, which may affect the stability of shore protection structures, and/or shorten their life. However, the future effects of sand supply reduction on the stability of shore protection structures is not included in this study.

3.7 MISCELLANEOUS

3.7.1 Traffic

Traffic on top of bulkheads may increase stresses within these structures. However, since the traffic is generally light near the top of shore protection structures, it was not considered in our evaluation.

3.7.2 Seismicity

Seismic activity imposes dynamic horizontal loads on shore protection structures and can be devastatingly large. This is an extremely important factor in some parts of the country, but the Lake Michigan area is relatively aseismic, and the potential for significant seismic damage to these structure is not large. Therefore, seismic effects were not included in this study.

4.0 STABILITY OF SHORE PROTECTION STRUCTURES

In this section, we evaluate the stability of the shore protection structures selected in Section 2.3. For each structure, we developed typical configurations and evaluated the effects of changes in water levels on the stability of these structures. In the following sections, we first describe the potential modes of failure for the different structures. Thereafter, for each structure, we provide a description of its configurations and our generalized engineering evaluations.

We note that the engineering evaluations consider several modes/causes of failure independently while in fact the changes of lake levels may affect more than one cause at the same time. For example, overtopping and toe scour may reduce the factor of safety against global stability and result in a more critical condition than if only toe scour is considered. The purpose of these evaluations is to provide only a qualitative evaluation of the effects on these structures due to changes in lake levels.

4.1 MODES OF FAILURE

There are several modes/causes of failure for shore protection structures including structural instability, global instability, wave runup/overtopping, piping/toe scour and flanking. A brief description of each of these modes is described below.

4.1.1 Structural Instability

Changes in lake levels may change the loading distribution on seawalls and bulkheads. For example, lowering lake levels is expected to increase stresses within a sheetpile/bulkhead due to a greater imbalance of forces on each side of the wall. Such excess stresses may overstress the sheetpile wall and result in failure by bending, anchorage destruction or interlock failure.

4.1.2 Global Instability

The design configurations of a bulkhead or a revetment are usually selected such that the soils retained are stable and the potential for a slope failure through the structure is minimal. Changes in lake level may affect the global stability of these walls. Again, lowering the lake level may reduce toe resistance and severely affect the global stability. Furthermore, larger wave forces associated with changing lake levels may cause groins to settle, slide or overturn.

4.1.3 Wave Runup & Overtopping

Wave runup and overtopping is usually associated with high lake levels and/or high waves, especially those with relatively long periods. The effect of overtopping is usually mitigated by placing drainage features in the structures that allow the water to drain away or through the structures without causing erosion and/or affecting the stability of the structure. If such drainage features are absent, erosion and increases in stresses against the structure may affect its stability. For example, the absence of a filter layer behind revetments can cause piping and the loss of the soil behind the wall.

4.1.4 Piping/Toe Scour

The buildup of water pressure behind structures will result in seepage through the structure or its foundation. Excessive seepage may cause loss of the retained soil or significant reduction in the shear strength of the soils located at the toe, which may result in the loss of foundation soils and failure of the structures.

Scour along the toe of seawalls/bulkheads and revetments may endanger their stability. Scour removes soils which normally buttress the toe of structures. Scour along the toe of the structures reduces the factor of safety against global stability and increases the stresses within the structure itself. This may result in a global instability and/or a structural failure of the seawalls/bulkheads or revetments.

4.1.5 Flanking

Flanking is the mode of failure involving wave actions eroding the soils behind the shore protection structures at either end. Even without changes in the lake level, flanking is a major mode of failure for bulkheads and revetments. However, increases in the lake levels are expected to accelerate failures due to flanking. Flanking is usually minimized by building wing walls along the sides or by connecting adjacent shoreline structures. The current structure inventories do not provide information on the use of wing walls or structural tie-ins between adjacent structures. Without such protection, all structures will fail due to flanking over time.

4.2 SEAWALLS/BULKHEADS

As stated previously, for this structure, we evaluated timber or steel vertical sheetpile walls, which are most common along the shore of the study area. Our evaluation is presented in the following sections.

4.2.1 Typical Configurations & Design Parameters

In general, the stability of a vertical cantilever sheetpile shore protection structure is affected by its height, depth of penetration, soil properties, water levels and wave characteristics. A brief description of each of these parameters and the basis for their selection is presented below. The typical configuration of the evaluated structure is presented in Figure 1 and summarized in Table 1.

FIGURE 1: TYPICAL CONFIGURATIONS OF CANTILEVER SEAWALL/BULKHEAD

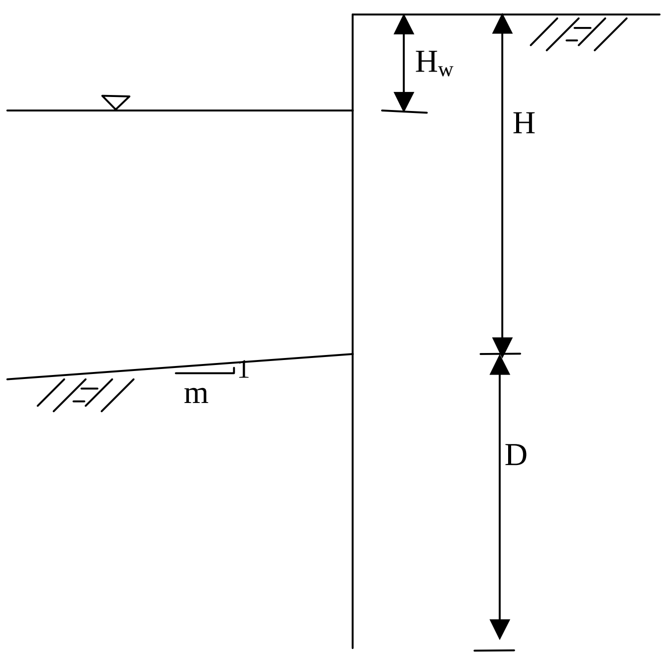

TABLE 1 DESIGN PARAMETERS SEAWALL / BULKHEAD STABILITY EVALUATION	
Structure Height (H)	1.52 m (5 ft)
Depth of Penetration (D)	Cohesive Soils: 3.35 m (11 feet) Cohesionless soils: 3.05 m (10 feet)
Soil Conditions	Cohesive Soils: Total Unit Weight = 19.2 kN/m^3 Cohesion = 0 Pa (Effective Stress Basis) Internal Friction Angle = 28° Cohesionless soils: Total Unit Weight = 19.2 kN/m^3 (122 pcf) Cohesion = 0 Pa Internal Friction Angle = 30°
Wall Friction	Cohesive Soils: 10° Cohesionless soils: 14°
Water Levels (H_w)	High: 0.46 m (1.5 feet) below the top of the structure Low: 1.52 m (5.0 feet) below the top of the structure
Lake Bed Slope (m)	20 h and 10 h on 1 v

4.2.1.1 Structure Height & Depth of Penetration – The structure height is a significant factor in its stability and is usually selected based on its use. An increase in height is associated with a significant increase in sustained stresses and cost. In general, for private recreation boat access (docking) purposes, the height ranges from 1.22 m (4 feet) to 1.52 m (5 feet). This same height is commonly found in the study area for shore protection purposes.

The depth of penetration of the piling usually depends on the sustained stresses which are associated with the structure height and type of retained soil. In general, the depth of penetration is usually selected to be approximately two times the wall height.

Based on a discussion with the USACE's Area Engineer in Grand Haven, Michigan, Detroit District, the length of most timber or steel sheetpiles used in seawalls along the Michigan shore of Lake Michigan is approximately 4.57 m (15 feet) with approximately 3.05 m (10 feet) of penetration depth (i.e., 1.52 m high walls). This length is usually limited by the maximum economical length of the available timber piles and the high cost of higher steel walls. For our evaluation, we selected a structure height of 1.52 m (5 feet).

Since cohesive soils are usually associated with higher stresses than cohesionless soils, for the 1.52 m high structure, we assumed that the depth of penetration in cohesive and cohesionless soils is 3.35 m (11 feet) and 3.05 m (10 feet), respectively.

4.2.1.2 Water Level – As discussed before, Lake Michigan levels have historically varied approximately 6 feet. Since the location of the water level against the structure is not known, based on discussions with the USACE's Grand Haven Area staff and discussions with W.F. Baird and Associates, we selected two water levels: 0.46 m (1.5 feet) and 1.52 m (5 feet) below the top of the sheetpile. Lower lake levels, below the 1.52 m level, will expose the structure and reduce the stresses against it. However, extreme low water levels are expected to increase the scour potential of soils at the toe of structures. Water levels of 0.46 m below the top of the structure and higher are expected to cause overtopping, which is the most critical condition for this type of structure, in addition to flanking. The effect of changing the water level from high to low or vice versa is evaluated in this study.

4.2.1.3 Wave Characteristics – As mentioned before, waves are characterized by their height and period. For the evaluation of the 1.52 m high sheetpiles, we considered one critical wave which is characteristic of Lake Michigan storms with a significant wave height of (Hs) of 5 m and a period (T) of 9 seconds.

4.2.2 Engineering Evaluations

4.2.2.1 Global Stability – Using the configuration shown in Figure 1, we estimated the factor of safety against global stability for the different soil, slope and water level conditions. The calculations were completed using a computer code called CWALSHT developed by the

USACE. The estimated factors of safety (FS) are summarized on Table 2, Global Stability Factors Of Safety/ Seawall/Bulkhead Shore Protection Structures.

TABLE 2 GLOBAL STABILITY FACTORS OF SAFETY SEAWALL / BULKHEAD STRUCTURES				
LAKE LEVEL (*) (m)	**COHESIVE SOILS**		**GRANULAR SOILS**	
	1:10 LAKE SLOPE	**1:20 LAKE SLOPE**	**1:10 LAKE SLOPE**	**1:20 LAKE SLOPE**
0.46	1.62	1.75	1.76	1.91
1.52	1.33	1.41	1.45	1.55
(*) Lake Level from top of Seawall/Bulkhead structure.				

The analyses were completed assuming that the water level is the same at both sides of the structure (long-term condition). As shown in Table 2, the FS for the typical seawalls/bulkheads ranges from 1.33 to 1.91. In general, a FS of 1.30 is considered acceptable for this type of structure, while a factor of safety of 1.50 is preferred.

As shown in Table 2, raising the lake level from 1.52 m to 0.46 m (as measured from the top of the structure) is expected to increase the factor of safety against global instability by approximately 20 percent. On the other hand, dropping lake levels will reduce the factor of safety. A review of the table indicates that flatter lake bed slopes are usually associated with a slightly higher factors of safety.

All factors of safety exceed the minimum acceptable value of 1.3 and are considered stable. Therefore, not considering other effects due to lower lake levels, including scour, the potential for a catastrophic global instability of an otherwise stable seawall/bulkhead due to a long-term or seasonal 1.06 m change in water level is small.

We also evaluated the potential for the lake level to drop ahead of the water table at landside. This condition is known as a rapid drawdown/seiche condition. It is most likely to affect cohesive soils where drainage within the cohesive soils lags the drop in water levels. The seiche event can be sustained for several hours but generally not longer than a day (typical storm event).

In this case, we assumed that the lake levels drop from 0.46 m to 1.52 m below the top of the structure and the water table within the retained soils stays at 0.46 m below the top of the structure. The results of our evaluation are presented on Table 3, Effects of Rapid Drawdown/Seiche Seawalls/Bulkheads Structures. As shown, the FS decreases significantly when the lake level drops from 0.46 m to 1.52 m below the top of the structure without drainage within the retained soils. The FS decreases from 1.62 to 1.04 and from 1.75 to 1.12 for the 1:20 and 1:10 lake slopes, respectively. Therefore, if the lake level drops rapidly from 0.46 m to 1.52 m as measured from the top of the structure, the structure may exhibit instability. Rapid drawdown or a seiche condition may trigger instability of shore protection sheetpile walls.

TABLE 3
EFFECTS OF RAPID DRAWDOWN/SEICHE
GLOBAL STABILITY FACTOR OF SAFETY
SEAWALL / BULKHEAD STRUCTURES

WATER TABLE (*) (m/m)	COHESIVE SOILS		GRANULAR SOILS	
	1:10 LAKE SLOPE	1:20 LAKE SLOPE	1:10 LAKE SLOPE	1:20 LAKE SLOPE
0.46/0.46	1.62	1.75	1.76	1.91
1.52/0.46	1.04	1.12	1.13	1.21

(*) 1.52/0.46: Water tables at lakeside/water table at landside.

The results of the rapid drawdown/seiche analyses can also apply to the dynamic condition of a wave against the structure. When the crest of a wave is against the structure, it will increase the stability of the structure because the water level at the lake side is higher than that on the land side. However, when the trough of a wave is against the structure, the water level at the lake side will be lower than that on the land side. This condition is similar to the rapid drawdown/seiche condition. For a wave with H_s of 5 m and T of 9 seconds as stated in Section 4.2.1.3, the depth of the trough will be more than 1.06 m (1.52 m – 0.46 m), which is the drop of the lake level analyzed in the rapid drawdown/seiche case. As stated previously, when the lake level drops quickly by 1.06 m, the structure may exhibit instability. When the lake level drops even lower due to a wave trough against a structure, instability of the wall may occur.

4.2.2.2 Wave Runup/Overtopping – We evaluated the effect of the critical wave described in Section 2 on the potential for runup and overtopping. Using the selected wave characteristics, we evaluated the runup potential for the two lake levels described before (i.e. 0.46 m and 1.52 m below the top of the structures).

We estimated the wave runup using the equations provided in *Design of Coastal Revetments, Seawalls, and Bulkheads* (USACE, 1995). The results of our evaluations indicate that water will overtop the sheetpile walls for both water levels. In other words, if the lake level is at 0.46 m or 1.52 m below the top of the structure, overtopping will occur.

If appropriate features are in place to drain the water such that it will not accumulate at the landside of a structure, the effects of overtopping may be mitigated. However, if these measures have not been implemented, overtopping may cause erosion and may affect the stability of the sheetpile structures. Soil behind the seawall/bulkhead becomes saturated and more susceptible to washout. Lakeward loads are also increased. We evaluated the potential effects of overtopping on the global stability of sheetpile walls when such drainage features are not in place and water accumulates on the landside of the structure.

In our evaluation, we assumed that the water level on the landside will reach the top of the structure for both lake levels. The results of the evaluation are summarized in Table 4, Effects of Overtopping Seawalls/Bulkheads Structures. Review of Tables 3 and 4, for the high lake level, indicates that the FS ranges from 1.62 to 1.91 without overtopping to 1.33 to 1.55 with overtopping. For the low lake level, the FS ranges from 1.33 to 1.55 without overtopping and from 0.90 to 1.05 with overtopping.

Therefore, overtopping is expected to reduce the factor of safety for global stability. For higher lake levels (0.46 m below the top of the structure), the FS of the structure is reduced by approximately 25 percent. However, the FS exceeds 1.3 and the structure is considered stable. When overtopping occurs at low lake levels, the FS of the structure is reduced by approximately 35 percent and the structure may exhibit instability.

TABLE 4
EFFECTS OF OVERTOPPING
GLOBAL STABILITY FACTOR OF SAFETY
SEAWALL / BULKHEAD STRUCTURES

LAKE LEVEL (*) (m)	**COHESIVE SOILS**		**GRANULAR SOILS**	
	1:10 LAKE SLOPE	**1:20 LAKE SLOPE**	**1:10 LAKE SLOPE**	**1:20 LAKE SLOPE**
0.46	1.33	1.44	1.43	1.55
1.52	0.90	0.97	0.97	1.05
(*) Lake Level below the top of Seawall/Bulkhead.				

4.2.2.3 TOE SCOUR – to evaluate the effect of toe scour on the stability of the structures, we estimated the lake bed elevation that will result in global instability (i.e., fs =1.0). In our evaluation, we assumed that no toe protection measures are in-place. The results of our evaluation are presented on table 5, effects of toe scouring/ seawalls/bulkheads structures. As shown in table 5, for low lake levels, instability is expected when the depths of toe scouring reach 0.61 m (2.0 feet) and 0.76 m (2.5 feet) for cohesive and granular soils, respectively. For high lake levels, 0.91 m (3 feet) of scour depth is required to trigger instability for both soils.

TABLE 5
EFFECTS OF TOE SCOURING
SEAWALL / BULKHEAD STRUCTURES

LAKE LEVEL (1) (m)	**DEPTH OF TOE SCOURING (m) (2)**	
	COHESIVE SOILS	**GRANULAR SOILS**
0.46	0.91	0.91
1.52	0.61	0.76

(1) Lake Level below the top of Seawall/Bulkhead.
(2) Toe scour required to cause global instability.

4.2.2.4 Structural Stability – To evaluate the structural stability, we estimated the maximum

moments within the cantilever sheetpile walls using the different soil and water levels discussed in Section 2.0. Based on the results of our evaluation, a drop in lake level from 0.46 m to 1.52 m below the top of the structure will increase the maximum moment by at least 23 percent. Rapid drawdown/seiche is expected to increase the maximum moment by approximately 50 percent or more.

We also evaluated the effect of overtopping on the maximum moment within the structure. Our analyses indicate that, for high lake levels (i.e. 0.46 m below the top of the structure) the maximum moments are expected to increase by approximately 23 percent or more. For the lower lake levels (i.e., 1.52 m below the top of the structure) overtopping is expected to increase the maximum moment by more than 50 percent.

Finally, we estimated the effect of scour. Our estimation indicates that the maximum moment may increase by at least 14 percent as a result of toe scour.

Because the potential for landward overturning of sheetpile structures is not large, assuming that retained soils are not washing out or are being eroded, dynamic effects (impact) from wave forces were not included in our evaluation.

In summary, changing lake levels are expected to have significant effects on the structural stability of sheetpile walls. Significant increases in maximum stresses are expected. Such increases may results in structural failures.

We note that some of the actions may occur at the same time. For example, lower lake levels and toe scour may occur at the same time. Thus, in some cases, the total increase in maximum moment will be higher than presented for each cause by itself. In addition, a sequence of high and low water levels may result in a combination of effects that will cause even higher increases in maximum moments.

4.3 REVETMENTS

As stated previously, we chose to evaluate riprap revetments, which are common along the shore of the study area. Our evaluation is presented in the following sections.

4.3.1 Typical Configurations & Design Parameters

In general, the stability of a revetment shore protection structure is dependent on its height, inclination, soil stratigraphy, water level, wave characteristics, armor unit, filter media, and lake bed slope. A brief description of each of these parameters and the basis for their selection is presented below.

4.3.1.1 Structure Heights & Inclinations – The height and inclination are significant factors in the stability of revetments. Based on discussions with the USACE's Grand Haven office, we evaluated two typical heights and two typical inclinations. We evaluated 1.52 m (5.0 feet) and 4.57 m (15.0 feet) high revetments with inclinations of three horizontal to one vertical and four horizontal to one vertical.

4.3.1.2 Water Level – We selected three water levels: at the toe of the structures and at one-third and two-thirds of the structure's height.

4.3.1.3 Armor Unit_ – The armor units of revetments are usually selected to sustain the effects of waves. The weight of the armor unit depends on the wave height and the inclination of the structure. For example, a larger wave height and a steep structure will need a heavier armor unit.

4.3.2 Engineering Evaluations

4.3.2.1 Armor Unit Stability – As the lake level rises, the depth of water increases and the breaking wave height increases, increasing the forces against the structure. On the other hand, reduction in the lake level may reduce the depth of water against the structure and cause a reduction in the height of the breaking waves.

We evaluated the effect of changing lake levels on the revetment armor units. The results of our evaluation indicate that higher lake levels will require heavier armor units. The breaking wave height for the lake water table at two-thirds of the structure is approximately twice that for the lake water table at one-third of the structure height. Therefore, when the lake level rises from one-third of the structure height to two-thirds of the structure height, heavier armor units will be needed for higher lake levels. The OTI inventory does not indicate the weight of armor stone.

4.3.2.2 Global Stability – Using the configurations stated previously, we estimated the factor of safety against global stability for the different water level conditions. The factor of safety was estimated using the Slices Method of Analyses. The Bishop simplified method was selected for this evaluation. The calculations were completed using a computer code called *PC-Slope* developed by Geoslope International. The estimated factors of safety (FS) are summarized on Table 6, Global Stability Factors of Safety/Revetment Shore Protection Structures.

TABLE 6
GLOBAL STABILITY FACTORS OF SAFETY
REVETMENT SHORE PROTECTION STRUCTURES

STEADY LAKE LEVEL	**1:3 STRUCTURE INCLINATION**		**1:4 STRUCTURE INCLINATION**	
	4.57 m STRUCTURE HEIGHT	**1.52 m STRUCTURE HEIGHT**	**4.57 m STRUCTURE HEIGHT**	**1.52 m STRUCTURE HEIGHT**
At Toe of Structure	1.7	1.8	2.0	2.1
At 1/3 Structure Height	1.5	1.7	1.9	2.1
At 2/3 Structure Height	1.5	1.9	1.9	2.3

The analyses were completed assuming that the water levels are the same at both sides of the structures (long-term condition). As shown in Table 6, the values of FS for the typical revetments range from 1.5 to 2.3. As expected, review of the table indicates that flatter structure slopes and smaller heights are usually associated with higher factors of safety.

Since the factor of safety for all structures under the different lake levels exceeds the required minimum of 1.3, changes in lake levels are not expected to have a significant effect on global stability of the evaluated revetments.

We also evaluated the potential effects of a lake level drop while the landward water table does not, which represents a rapid drawdown/seiche case. For this case, we assumed that the lake level had dropped to the toe of the structure and the water table was at 1/3 the height of the structure at landside. The results of our evaluation are presented in Table 7, Effects of Rapid Drawdown/Seiche Revetments Structures. For comparison purposes, the factors of safety for the structures before and after rapid drawdown/seiche are presented in Table 7. As shown, the factor of safety decrease when the lake level drops from 1/3 of the structure height to the toe of the structure. The drop in the FS ranges from approximately 35% for the 4.57 m structure to approximately 19% for the 1.52 m structure. As shown, the factors of safety drop below 1.3 which may result in instability.

TABLE 7
EFFECTS OF RAPID DRAWDOWN/SEICHE
GLOBAL STABILITY FACTOR OF SAFETY
TYPICAL REVETMENT STRUCTURES

WATER LEVEL (*)	1:3 STRUCTURE INCLINATION		1:4 STRUCTURE INCLINATION	
	4.57 m STRUCTURE HEIGHT	1.52 m STRUCTURE HEIGHT	4.57 m STRUCTURE HEIGHT	1.52 m STRUCTURE HEIGHT
At 1/3 Structure Height / At 1/3 Structure Height	1.5	1.7	1.9	2.1
At Toe of Structure / At 1/3 Structure Height	1.0	1.4	1.2	1.7

(*) Water tables at lakeside/water table at landside.

Furthermore, we also evaluated the effect of ice on the stability of the revetment. The ice loading occurs when the frozen lake level drops, which causes the weight of ice to be exerted on the revetments. In our evaluation, we assumed that the frozen lake level dropped from 1/3 of structure height to the toe of the structure. Due to the drop of the lake level, ice accumulated on the lower surface of the revetment. Our evaluations are presented on Table 8, Effects of Ice Force on Revetment Structures. As shown on Table 8, the presence of the ice on the lower surface of the revetment will have minimal effect on the stability of the evaluated revetments. This is due to the fact that ice accumulation occurs at the lower portion of the structure, thus, increasing the resistance for sliding. We note that ice damage may still occur due to lateral loading or displacement of the armor units.

TABLE 8 EFFECTS OF ICE FORCES GLOBAL STABILITY FACTOR OF SAFETY REVETMENT STRUCTURES				
	1:3 STRUCTURE INCLINATION		1:4 STRUCTURE INCLINATION	
	4.57 m STRUCTURE HEIGHT	1.52 m STRUCTURE HEIGHT	4.57 m STRUCTURE HEIGHT	1.52 m STRUCTURE HEIGHT
No Ice Force	1.7	1.8	2.0	2.1
Ice Force	1.7	2.0	2.1	2.3
Note: Water tables at toe of the structure at both sides of the structure.				

4.3.2.3 Effects of Wave Runup/Overtopping – To evaluate the potential effects of wave runup on our revetments, we used the wave presented in Section 2. We evaluated the runup potential against our typical revetment structures with the assumed lake levels stated previously. The results of our evaluations indicate that the water is likely to overtop both of the revetment structures evaluated herein.

If appropriate features are in place to drain the water such that it will not accumulate at the landside of a structure, the effects of overtopping may be mitigated. However, if these measures have not been implemented, overtopping may cause erosion and may affect the stability of the revetment structures. Soil behind the revetment becomes saturated and more susceptible to washout.

Lakeward loads are also increased. Piping will occur if proper filter media is not in-place and will undermine the back of the revetments causing it to fail structurally. In our evaluation, we assumed that appropriate filter layer and drainage features were in place. No information is in the OTI survey regarding the filter layer and drainage features behind the revetments inventoried.

4.3.2.4 Toe Scour – Riprap revetments are usually protected against scour by placing stone along the toe. In our evaluation, we assumed that the toe protection of revetments was designed in accordance to the design procedures provided in *Design of Coastal Revetments, Seawalls, and Bulkheads* (USACE, 1995). Therefore, the potential for scour is minimized and is expected to occur only slowly over a long period of time. The OTI inventory does not indicate whether toe protection is present or not.

4.4 GROINS

As stated previously, for this structure type, we evaluated concrete grout-filled geotextile groins which are common along the shore of the study area. Our evaluation is presented in the following sections.

4.4.1 Typical Configurations & Design Parameters for Groins

In general, the stability of a groin is affected by its height, its unit weight, the water level and wave characteristics. A brief description of each of these parameters and the basis for their selection is presented below.

4.4.1.1 Height, Length & Configuration – We selected the groin shape and configuration based on our discussions with the USACE's staff in the Grand Haven Area Office. In our evaluation, each groin consists of 3 oval-shaped tubes. While dimensions vary greatly, we selected a groin size as follows. Each tube is about 0.61 m (2 feet) wide by 0.46 m (1.5 feet) in height. The groins are arranged such that two tubes are at the bottom and one tube at the top, for a total bottom width of 1.2 m (4 feet) and total height of 0.9 m (3 feet). The length of the groins is approximately 18.29 m (60 feet) from the water line.

4.4.1.2 Unit Weight – The major mode of failure of grout-filled groins (geobags) is undermining of the bags resulting in differential settlement and fracture of the grout tube. The fractured pieces of the tube are then moved around by wave action. The unit weight of the groin per linear foot of groin plays an important role in its stability. Heavier materials provide higher resistance to sliding and overturning. For this evaluation, the unit weight of the concrete grout was selected as 23.6 KN/m^3 (150 pcf), the unit weight of concrete.

4.4.1.3 Water Level & Wave Characteristics – In our evaluation we used the deepwater wave presented in Section 2.

There are three basic types of waves: non-breaking, breaking and broken waves. Because groins are usually structures perpendicular to the shoreline, they may be subjected to these three types of waves. In deep waters, depending on how long the groin is, the waves acting against the groin are non-breaking. As they move closer to shore, there is a point at which the waves start to break and a portion of the groin is subjected to the action of breaking waves. As they keep moving toward the shore, all the waves are broken as they collapse against the groin. Breaking waves produce the largest dynamic force and moment against a structure.

Changes of lake level will change the distribution of forces along the groin. As lake levels increase, more of the groin length may be subjected to non-breaking waves and less to broken waves. At some point, the groins may be subjected only to non-breaking waves. However, high lake levels may results in flanking at the shore end of the groin. On the other hand, as lake levels drop, more of the groin length may be subjected to breaking and broken waves.

To evaluate the effects of lake level changes on the stability of the selected groin configurations, we assumed a maximum of 1.8 m (6 feet) of change in the lake level. This change will cover the historical range of change in the Lake Michigan levels. In addition, we assumed that the still water level is at the bottom of the groin's land-end.

4.4.2 Engineering Evaluation

To estimate the wave forces acting on the groin, we estimated the depths at which the wave will break and the breaker height. The selected wave (i.e., Hg = 5.0 m, T = 9 sec) will break approximately at a depth of 6.10 m (20 feet) for lake bottom slopes of 1:10 and 1:20, which corresponds to distances from the shore of 61.0 m (200 feet) and 122 m (400 feet), respectively. In addition, the breaker height will be approximately 6.10 m (20 feet). As a result, the 60-feet long groins will be subjected only to broken waves.

We evaluated the forces acting on the groin from broken waves based on equations provided in the Shore Protection Manual (USACE, 1984). The equations assumed that the broken wave pressure is uniformly distributed from the still-water level to a height, hc, equal to 3/4 of the height of a breaking wave. Therefore, only the above-water portion of the groin will be subjected to the dynamic pressure. Then we estimated the factor of safety against sliding and overturning.

The results of our evaluation indicate that, for the selected wave characteristics, the groins with the unit weigh of 23.6 KN/m^3 (150 pcf) will fail by lateral displacement and overturning under broken wave forces. Although the height of the wave pressure is approximately 4.57 m (15 feet), only 0.91 m (3 feet), or the height of the groin, of that pressure is acting against the groin above the still water level. Therefore, a rise of lake level by 6 feet will reduce the dynamic force against the groin due to a decrease in the above-water portion of the groin. On the other hand, dropping lake levels are expected to increase the dynamic broken wave force against the groin due to the increase in the above water portion of the groin. However, a drop of six feet in lake water level will expose the entire length of the groin. In this case, the groin will not be subjected to any wave force.

5.0 CONCLUSIONS

As presented in Section 4, changing lake levels are expected to have some effects on the evaluated structures. Rising and falling water levels have different effects on the stability of these structures; however, cycles of rising and falling water levels may result in a complex combined effect.

For example, low levels may increase toe scour, reducing the toe resistance for sheetpile wall bulkheads, and high water levels increase the likelihood of overtopping which may increase stresses on the structure.

We note that the effects described here are associated with the evaluated structures under the selected wave characteristics and changes in the lake levels. More severe wave characteristics and lake level changes may have even more severe effects. In this section, we summarize the potential effects evaluated in this study, and present our conclusions. In addition, the failure potential for the evaluated structures are summarized in Tables 9 and 10.

Based on the evaluation completed in this study, summarized in Tables 9 and 10, and engineering judgement, we developed an estimate of the percentage loss of the values of the shore protection structures over a 50-year period. Our estimation is presented in Table 11. This estimate was prepared to aid in the development of a cost estimate for the potential damage due to potential changes in Lake Michigan levels and should be used for this purpose only. The estimates are for privately-owned structures only. It is assumed for this study that publicly-owned structures were adequately designed, constructed, and maintained.

5.1 SEAWALLS/BULKHEADS

It is clear that a reduction in lake level affects global stability of these structures. Low levels are associated with low toe resistance and high scour rates. The results of this study indicate that low lake levels may result in a reduction of approximately 20 percent in the factor of safety against sliding. If low lake levels persist, toe scour will further reduce the factor of safety.

Drawdown/seiche of approximately 3.5 feet may be associated with global instabilities along the shorelines and will not be limited to areas with protection structures. The results of this study indicate that, based on the assumptions in our analyses, a reduction in the factor of safety due to drawdown/seiche may exceed 30 percent.

Low lake levels reduce the potential for overtopping. However, if overtopping occurs, when the lake level is low, due to high waves, the additional stresses on the structure combined with the reduction in toe resistance associated with the low levels may result in instability.

Rising lake levels have a mixed effects on the stability of seawalls and bulkheads. A 1.06 m rise in lake level is expected to increase the factor of safety against global stability by approximately 20%, due to an increase in the toe resistance, and decrease the factor of safety by approximately 18% to 33%, due to an increase in the stresses on the structure assuming overtopping occurs with insufficient drainage features in place. Again, mitigation of overtopping effects will minimize the effects of rising lake levels on these structures.

Flanking continues to be a major failure mode. It is essential that measures be implemented to minimize the potential for flanking. All isolated structures that lack wingwalls will fail due to flanking in a 50-year period.

In summary, we conclude that, while we have no influence on the lake levels, it appears that mitigating overtopping effects by installing drainage features and providing toe scour protection and edge flanking protection may minimize the potential for failure and increase the useful life of the structures. Structures without these features have a high probability of failure.

As stated previously, we developed an estimate of the percentage loss of the values of the shore protection structures over a 50-year period, which is presented in Table 11. As shown on Table 11, the estimated loss of structure values for the seawalls/bulkheads in a 50-year period ranges from 35 percent to 100 percent for a extreme low lake level and from 50 percent to 100 percent for a extreme high lake level.

5.2 REVETMENTS

Falling lake levels are expected to adversely affect the global stability of revetments. However, the factors of safety of the structures evaluated in this study are satisfactory for the range of lake levels assumed. In addition, the potential for adverse effects due to toe scour is less at low lake levels because riprap revetments usually include toe protection. However, armor stability is expected to be affected by rising lake levels. Rising lake levels from one-third to two-thirds the height of the structure will double the breaker height, which may damage the armor units.

Again, rapid drawdown/seiche may cause instability if drainage features are not in-place. Flanking is an important failure mode and rising lake levels are expected to accelerate flanking.

Rising lake levels may also result in overtopping. The effect of overtopping on revetments is expected to be small since riprap revetments usually allow for rapid drainage. This is assuming that the construction of these revetments included an adequate filter layer.

As shown on Table 11, the estimated loss of structure values for the seawalls/bulkheads in a 50-year period ranges from 10 percent to 75 percent for a extreme low lake level and from 75 percent to 90 percent for a extreme high lake level.

5.3 GROINS

Groins evaluated in this study will be subjected to broken waves. These groins are expected to fail under broken wave conditions for the design wave. Rising lake levels will reduce the dynamic force against the groin due to a decrease in the above-water portion of the groin. On the other hand, falling lake levels are expected to increase the dynamic broken wave force against the groin due to the increase in the above-water portion of the groin. We estimated, as shown in Table 11, that the groin will lose 100 percent of its structure value during a 50-year period.

6.0 REFERENCES

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Personal Communication with Mr. Ross Kittleman, USACE Area Engineer in Grand Haven, Michigan, Detroit District.

TABLE 9 SUMMARY OF STABILITY EVALUATION AND POTENTIAL FOR FAILURE SEAWALL / BULKHEAD STRUCTURES							
Failure Mode	**Mechanism**	**LAKE LEVEL**	**WATER TABLE AT LANDSIDE**	**LAKE BED SLOPE**	**SOIL TYPE**	**CHANGE IN FS**	**POTENTIAL FOR FAILURE**
Global Stability	Rising water table at both sides from 1.52 m to 0.46 m	0.46 m	Same as lake level	1:10 and 1:20	Cohesive and granular soils	+20%	Very Low
	Dropping water table at both sides from 0.46 m to 1.52 m	1.52 m	Same as lake level	1:10 and 1:20	Cohesive and granular soils	-20%	Very Low
	Rapid Drawdown: Lake level from 0.46 m to 1.52 m	1.52 m	0.46 m	1:10 and 1:20	Cohesive soils	-32%	High
Runup/Overtopping	Runup/Overtopping	1.46 m	0.00 m	1:10 and 1:20	Cohesive soils	-18%	Low
		1:52 m	0.00 m	1:10 and 1:20	Cohesive soils	-33%	High
Toe Scour	Toe Scour: 0.91 m scour depth	0.46 m	Same as lake level	1:10 and 1:20	Cohesive soils	-38% ~- 43%	High
	Toe scours: 0.61 m scour depth	1.52 m	Same as lake level	1:10 and 1:20	Cohesive soils	-25% ~- 29%	High
Flanking	Structures not protected from flanking using wing walls or connection to adjacent structures will fail.						High

Notes:
1 Seawall/bulkhead structures have a height of 4.57 m with penetration depths ranging from 3.05 m to 3.35 m.
2. Lake level and water table are measures below the top of structure.
3. This table presents several modes/causes of failure independently while in fact the changes of lake levels may affect more than one cause at the same time and result in a more critical condition.

TABLE 10
SUMMARY OF STABILITY EVALUATION AND POTENTIAL FOR FAILURE
REVETMENT STRUCTURES

FAILURE MODE	MECHANISM	STRUCTURE HEIGHT	STRUCTURE INCLINATION	LAKE LEVEL	WATER TABLE AT LAND SIDE	CHANGE IN FS	POTENTIAL FOR FAILURE
GLOBAL STABILITY	Rising lake level from 1/3 structure height to 2/3 structure height	4.57 m	1:3 and 1:4	1.52 m	Same as lake level	0%	Low
		1.52 m	1:3 and 1:4	0.51 m	Same as lake level	+10%	Low
	Dropping lake level from 1/3 structure height to toe of structure	4. 57 m	1:3	4. 57 m	Same as lake level	+13%	Low
		4. 57 m	1:4	4. 57 m	Same as lake level	+5%	Low
		1.52 m	1:3	1.52 m	Same as lake level	+6%	Low
		1.52 m	1:4	1.52 m	Same as lake level	0%	Low
	Rapid drawdown: Dropping lake level from 1/3 structure height to toe of structure	4. 57 m	1:3	4. 57 m	3.0 m	-33%	High
		4. 57 m	1:4	4. 57 m	3.0 m	-37%	High
		1.52 m	1:3	1.52 m	1.0 m	-18%	Low
		1.52 m	1:4	1.52 m	1.0 m	-25%	Low
ARMOR STABILITY	Rising lake level from 1/3 structure height to 2/3 structure height	4.57 m and 1.52 m	1:3 and 1:4	1.52 m and 0.51 m	Same as lake level	Not Applicable	High
RUNUP / OVERTOPPING	Structures with appropriate drainage features to minimize the potential for water accumulation at landside						Low
TOE SCOUR	Structures with appropriate toe protection						Low
FLANKING	Structures not protected from flanking using wing walls or connection to adjacent structures will failure						High
ICE FORCE	Frozen lake level dropped from 1/3 structure height to the toe of the structure and ice accumulated on the lower surface of the structures.	4.57 m, and 1.52 m	1:3 and 1:4	4.57 m, and 1.52 m	Same as lake level	0% ~ +10%	Low

Notes:
1. Analyzed revetment structures have heights of 4.6 m and 1.5 m with slope inclinations of 1:3 and 1:4 (vertical:horizontal).
2. Lake level and water table are measured from top of structure.
3. This table presents several modes/causes of failure independently while in fact the changes of lake levels may affect more than one cause at the same time and result in a more critical condition.

TABLE 11
EVALUATION OF POTENTIAL DAMAGES TO PRIVATELY-OWNED SHORE PROTECTION STRUCTURES OVER 50-YEAR STUDY PERIOD

STRUCTURES	LAKE LEVEL	STRUCTURE TYPE	POSSIBLE FAILURE MODE	% LOSS OF STRUCTURE VALUE
Seawalls / Bulkheads	Extreme Low	Timber Structures	Deterioration	100
		Continuous Steel Structures	Scouring and Overtopping	35
		Isolated Steel Structures	Flanking, Scouring, and Overtopping	80
	Extreme High	Timber Structures	Deterioration	100
		Continuous Steel Structures	Rapid Drawdown and Overtopping	50
		Isolated Steel Structures	Flanking, Rapid Drawdown, and Overtopping	90
Revetments	Extreme Low	Continuous Structures	Armor Unit Displacement and Rapid Drawdown	10
		Isolated Structures	Flanking, Armor Unit Displacement, and Rapid Drawdown	75
	Extreme High	Continuous Structures	Armor Unit Displacement and Rapid Drawdown	75
		Isolated Structures	Flanking, Armor Unit Displacement, and Rapid Drawdown	90
Tube Groins	Extreme Low and High	Single and Multiple	Settlement, Scour, and Wave Action	100

APPENDIX A

Summary of OTI Survey